Non-Linear Dynamic Alternate Path Analysis for Progressive Collapse: Detailed Procedures Using UFC 4-023-03 (revised July 2009)

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Abstract

In July of 2009, The US Department of Defense (DoD) released the newly revised version of the UFC 4-023-03 "Design of Buildings to Resist Progressive Collapse." The new UFC incorporates both direct and indirect design procedures. The main direct design procedure is the Alternate Path (AP) method, in which a structure is analyzed for collapse potential after the removal of a load bearing vertical component. Different analytical procedures may be used for AP, including Linear Static (LS), Nonlinear Static (NLS), and Nonlinear Dynamic (NLD). Significant changes have been made to AP methods for analysis in the new criteria which result in less conservative and more efficient structural components.

Typically, when doing AP analyses, designers often choose static procedures which tend to be simpler, requiring less labor. However, progressive collapse is a dynamic and nonlinear event, and the load cases used for the static procedures that account for inertial and nonlinear effects tend to add conservatism to final design. The procedures in the criteria, from NLS to NLD, offer analysis procedures of increasing complexity and time investment, but offer a significant return of increased design efficiency. This paper presents a detailed example of how to properly perform an NLD AP analysis on a building following the guidelines in the new UFC 4-023-03 and the advantages in the final design when compared to the results of an LS analysis.

Alternate Path Analysis per UFC 4-023-03

In the Alternate Path (AP) method, the designer must show that the building is capable of bridging over a removed load-bearing structural element and that the resulting extent of damage does not exceed the acceptance criteria contained in the guidelines. Three different analytical procedures may be used: Linear Static (LS), Nonlinear Static (NLS), and Nonlinear Dynamic (NLD). The guidelines also provide specific information regarding the extent, location and number of column removals that must be performed to successfully demonstrate that the structure is able to resist progressive collapse. The example presented in this paper focuses on the application of the NLD procedure for AP analysis for one column removal of a middle perimeter column in a decades old concrete building being updated for DoD occupancy as shown in Figure 1.

Report Documentation Page

Form Approved
OMB No. 0704-0188

1. REPORT DATE **JUL 2010**	2. REPORT TYPE **N/A**	3. DATES COVERED **-**
4. TITLE AND SUBTITLE **Non-Linear Dynamic Alternate Path Analysis for Progressive Collapse: Detailed Procedures Using UFC 4-023-03 (revised July 2009)**		5a. CONTRACT NUMBER 5b. GRANT NUMBER 5c. PROGRAM ELEMENT NUMBER
6. AUTHOR(S)		5d. PROJECT NUMBER 5e. TASK NUMBER 5f. WORK UNIT NUMBER
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) **Protection Engineering Consultants**		8. PERFORMING ORGANIZATION REPORT NUMBER
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)		10. SPONSOR/MONITOR'S ACRONYM(S) 11. SPONSOR/MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release, distribution unlimited

13. SUPPLEMENTARY NOTES
See also ADM002313. Department of Defense Explosives Safety Board Seminar (34th) held in Portland, Oregon on 13-15 July 2010, The original document contains color images.

14. ABSTRACT
In July of 2009, The US Department of Defense (DoD) released the newly revised version of the UFC 4]023]03 gDesign of Buildings to Resist Progressive Collapse.h The new UFC incorporates both direct and indirect design procedures. The main direct design procedure is the Alternate Path (AP) method, in which a structure is analyzed for collapse potential after the removal of a load bearing vertical component. Different analytical procedures may be used for AP, including Linear Static (LS), Nonlinear Static (NLS), and Nonlinear Dynamic (NLD). Significant changes have been made to AP methods for analysis in the new criteria which result in less conservative and more efficient structural components. Typically, when doing AP analyses, designers often choose static procedures which tend to be simpler, requiring less labor. However, progressive collapse is a dynamic and nonlinear event, and the load cases used for the static procedures that account for inertial and nonlinear effects tend to add conservatism to final design. The procedures in the criteria, from NLS to NLD, offer analysis procedures of increasing complexity and time investment, but offer a significant return of increased design efficiency. This paper presents a detailed example of how to properly perform an NLD AP analysis on a building following the guidelines in the new UFC 4]023]03 and the advantages in the final design when compared to the results of an LS analysis.

15. SUBJECT TERMS

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a. REPORT **unclassified**	b. ABSTRACT **unclassified**	c. THIS PAGE **unclassified**	**SAR**	**37**	

UFC 4-023-03: Differences between 2005 and 2009 versions

In the four years since Unified Facilities Criteria (UFC) 4-023-03 Design of Buildings to Resist Progressive Collapse was first published in January of 2005, various omissions, ambiguities, and opportunities for improvement were identified by civilian and government designers and engineers. A significant revision to the Progressive Collapse UFC was initiated in the Fall of 2006 and was completed in 2009, during which a number of significant improvements were made. Occupancy Categories (OCs) similar to those in ASCE 7 Minimum Design Loads for Structures are used to define a building's progressive collapse design requirements; previously, military definitions of levels of protection were used. Indirect design methods for enhancing load redistribution capacity with tie forces and direct design methods using alternate path and specific local resistance continue to be employed but with revisions. The alternate path method now includes dynamic and load increase factors that are based on careful analysis of the inertial and nonlinear aspects of load redistribution. Structural response criteria are specified in terms of force- and deformation-controlled actions, similar to ASCE 41-06 Seismic Rehabilitation of Existing Buildings. A non-threat specific, local hardening procedure was developed and implemented, to insure ductile behavior of critical elements without significant additional cost, for new construction. Finally, a brief overview of three example problems is provided.

Building Description

The building analyzed is a four story reinforced concrete (RC) building built in the 1940's. The main floor system is composed of a two way flat slab system with drop panels and corbels around the columns. There are also one-way slabs and beams, including a continuous perimeter beam around the building. Typical bay spacing around the building is twenty feet in the east-west direction and twenty five feet in the north-south direction. The average floor to floor height is 11 feet. Typical sizes of the structural components are as follows:

- Typical spandrel beam: 12 in x 30 in
- Typical slab: 9 in
- Typical perimeter columns: 24 in x 24 in

Based on available information, the material properties used are as follows. When materials were not available, their values were estimated using the values recommended in the ASCE Standard *Seismic Rehabilitation of Existing Building* (ASCE 41-06) to be as follows:

- RC beams and slab - design compressive strength (f'_c) = 3500 psi
- RC columns - design compressive strength (f'_c) = 4500 psi
- Steel reinforcement yield strength (f_y) = 40000 psi

Per ASCE 41-06 and as allowed in UFC 4-023-03, over-strength factors for concrete compressive strength of 1.5 and steel reinforcement yield strength of 1.25 were used in the analysis.

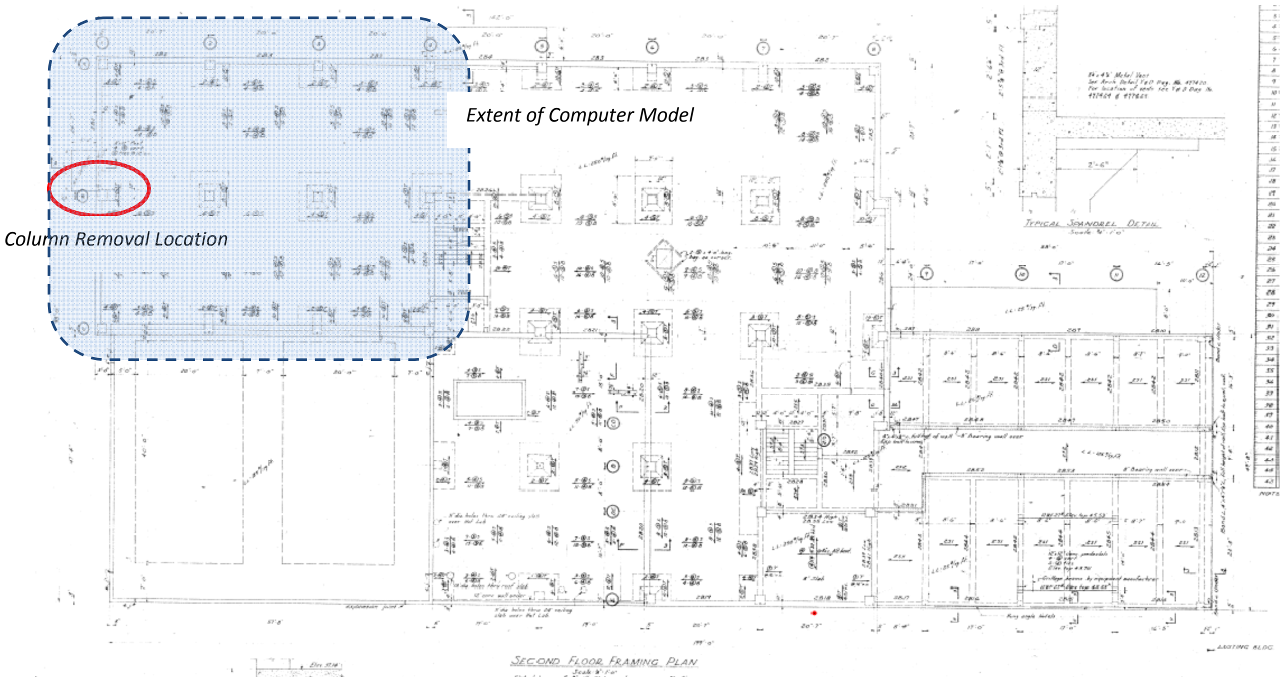

Figure 1. Building Floor Plan

Non-linear Dynamic (NLD) AP Analysis Procedure

In the non-linear dynamic procedure, the un-factored load case (extreme load event case) is directly applied to a materially and geometrically nonlinear model of the structure. In the first phase of the dynamic analysis, the structure is allowed to reach equilibrium under the applied load case. In the second phase, the column or wall section is removed almost instantaneously and the software tool calculates the resulting motion of the structure. The resulting maximum member deformations are then compared to the deformation limits for deformation controlled actions and forces for force-controlled actions that are provided in the guidelines, which are dependant of construction type and structural configuration. If the deformation limits are exceeded at any hinge locations, the deficient structural components are re-designed and the analysis is re-run until no deformation limits are exceeded at the hinge locations. The application of this procedure is presented step by step in the following sections.

Step 1 – Development of Computer Model

A 3-dimensional computer model of the building was built using the conventional structural design software SAP2000. This structural design software has the capability of modeling dynamic load cases and non-linear component response. To simplify the analysis and reduce running time, only the north-west portion of the building where the column removal is located was modeled (See Figure 1). Geometry, structural configuration and material properties are all incorporated into the model at this stage as explained in the following paragraphs.

Building components: As required in UFC 4-023-03, the primary components were included explicitly in the model of the structure. Primary components are defined in the UFC 4-023-03 as any components that contribute to the resistance of the collapse. Primary components included were columns, spandrel beams, and column strips in the two-way slabs. Secondary components are elements that do not significantly contribute to the resistance of the structure. The model did not explicitly include the secondary components but the effects of the secondary components in the load path and load distribution was taken into consideration when determining the linear loads applied to the primary members. This is explained in more detail under “Loads” below. Secondary components were also checked against the allowable deformation limits as required in the guidelines.

The columns were modeled with line elements as axial components pinned at the foundation and continuous at floor levels. Non-linear hinges were not added to column components. Spandrel beams were modeled with line elements as flexural components with steel reinforcement per the structural drawings. Non-linear hinges where added at the ends and middle span of every beam element. To account for the added capacity provided by the floor slab, the column strip of the slab system was modeled explicitly as a beam component (See Figure 3 below). As with the spandrel beam, steel reinforcement for the column strip of the two-way slab system was determined per the information available in the structural drawings. Non-linear hinges were also added at the ends and middle span of every column strip. Determination of hinge type and response criteria will be shown later in more depth.

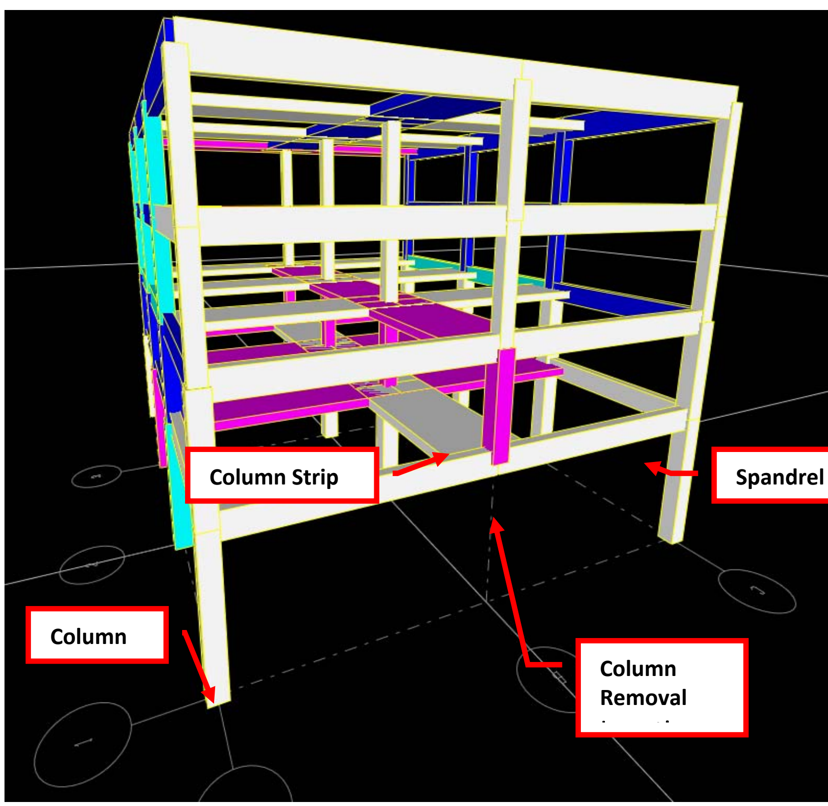

Figure 2. Computer Model

Steel Reinforcement in Beams and Slabs: Depending on the location of a given beam component with respect to the loss location, the "effective reinforcing steel" in the high moment regions varies. Therefore, adequate interpretation of the expected response of a given component was required in order to properly assign the amounts of steel reinforcement for that particular component in the model. For example, the steel reinforcement in the spandrel beams of this building consist of bent bars, straight bars, and top bars as shown in Figure 3. For beams whose effective span remains unchanged after the column removal, the expected high moment regions will be at the ends (negative moment) and at mid-span (positive moment). Therefore, the negative region steel reinforcement assigned to these beam elements corresponded to the top bars plus the bent bars, and the effective positive region reinforcing steel corresponded to the straight bars only.

This differed at the column loss location. The loss of the column would effectively change two short beams into one long beam component. Therefore, the positive moment region for this case now includes the column joint at the removal location. In this case, the amount of positive steel must be adjusted to account for the effective area of bottom bars running across the joint, which depends of the bars overlap provided. If the straight (bottom) bars are not overlapped at the joints as often was the case in older designs, the beam component has no effective positive moment capacity. The amount of effective bottom steel assigned to the spandrel beams at the loss locations thus included a reduction factor based on the splice length provided compared to the required splice length per the ACI design code.

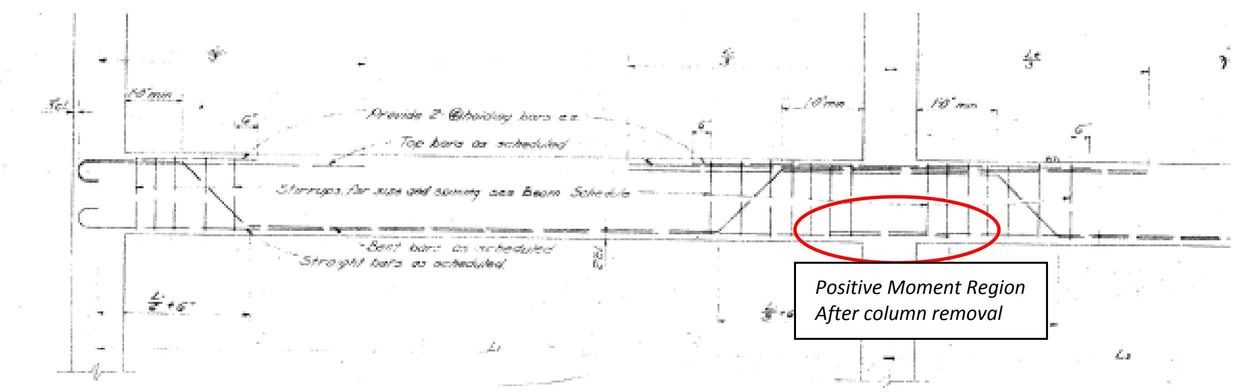

Figure 3. Beams Steel Reinforcement Layout

Step 2 – Loads and Mass

The analysis was performed using the extreme load combination of 1.2DL + 0.5L as specified in UFC 4-023-03. The loads used were as follows:

- Floor Dead Load (DL) 112 psf (2nd floor), 93.75 psf (all other)
- Floor Live Load (LL) 125 psf
- Perimeter Beam + Parapet (per typical spandrel detail) 875 lbf/ft

Since the building model only included primary components (beams, slab column strips and columns), the load applied to the beams and columns strips was calculated manually and applied to each component as a linear load. The linear load corresponding to each component was calculated based on the component's tributary area as determined from the building plans. An example of the calculation of the linear loads applied to beam components at the second floor level is presented next along with Figure 4 to help illustrate the approach.

Dead Load $\mathrm{DL} := 150\,\mathrm{pcf}\cdot 9\,\mathrm{in} = 112.5\,\mathrm{psf}$

Live Load $\mathrm{LL} := 125\,\mathrm{psf}$

Total Load $q := 1.2\,\mathrm{DL} + 0.5\,\mathrm{LL} = 197.5\,\mathrm{psf}$

Perimeter load (beam + parapet typical 2nd floor) $wp := 150\,\mathrm{pcf}\cdot 12\,\mathrm{in}\cdot 70\,\mathrm{in} = 875\cdot\frac{\mathrm{lbf}}{\mathrm{ft}}$

Linear Load on Column Strips

$$w_{csE\_W} := 2\cdot 0.25\cdot\frac{q\cdot 20.58\,\mathrm{ft}\cdot 24.58\,\mathrm{ft}}{20.58\,\mathrm{ft}} = 2.43\cdot\frac{\mathrm{kip}}{\mathrm{ft}}$$

$$w_{csN\_S} := 2\cdot 0.25\cdot\frac{q\cdot 20.58\,\mathrm{ft}\cdot 24.58\,\mathrm{ft}}{24.58\,\mathrm{ft}} = 2.03\cdot\frac{\mathrm{kip}}{\mathrm{ft}}$$

Linear Load on Perimeter Beams

$$w_{pbN\_S} := 0.25\cdot\frac{q\cdot 20.58\,\mathrm{ft}\cdot 24.58\,\mathrm{ft}}{24.58\,\mathrm{ft}} + 1.2\cdot wp = 2.07\cdot\frac{\mathrm{kip}}{\mathrm{ft}}$$

$$w_{pbE\_W} := 0.25\cdot\frac{q\cdot 20.58\,\mathrm{ft}\cdot 24.58\,\mathrm{ft}}{20.58\,\mathrm{ft}} + 1.2\cdot wp = 2.26\cdot\frac{\mathrm{kip}}{\mathrm{ft}}$$

To apply the loads to the structure, separate load patterns for each of the load types (Dead, Live, Perimeter, etc.) were created and assigned to the corresponding beam elements as linear loads. This is illustrated in Figure 5 below.

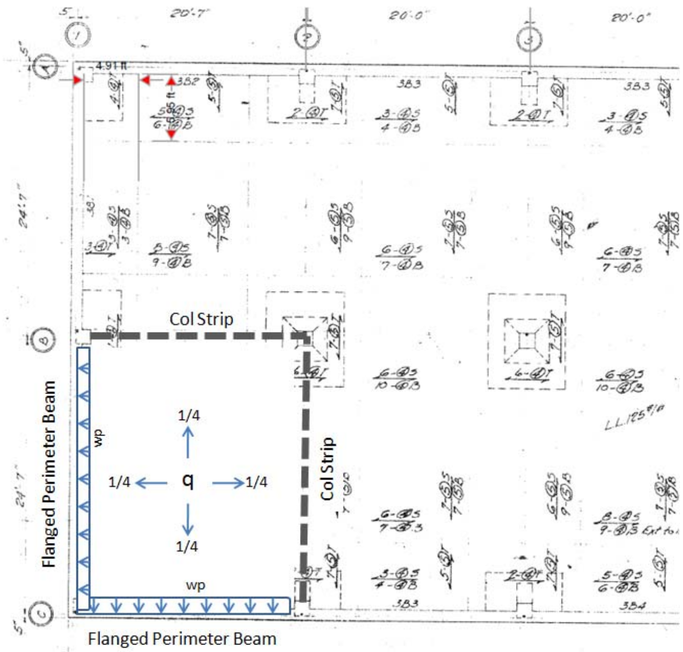

Figure 4. Linear Load Distribution Calculation

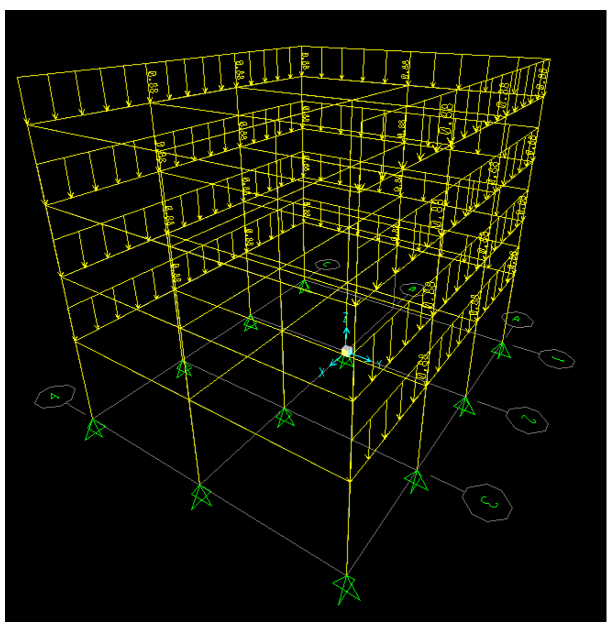

Figure 5. Loads Applied to Building as Linear Loads (Perimeter Load)

The linear dead loads calculated for the spandrel beams and slabs column trips (shown above) included the self weight of these components, which constitute the mass of the dynamic system. Therefore, in order to properly calculate the mass for the analysis, the computer model was set to use the added dead loads (instead of the elements) as the mass source.

Step 3 - Non-linear Hinges

Non-linearity was incorporated into the analysis by using non-linear flexural hinges placed in the beam components. These hinges are placed at the ends and mid-span of all beam elements, since these are the expected "high flexural stress" regions in the beam. See Figure 6 for hinge locations in the model

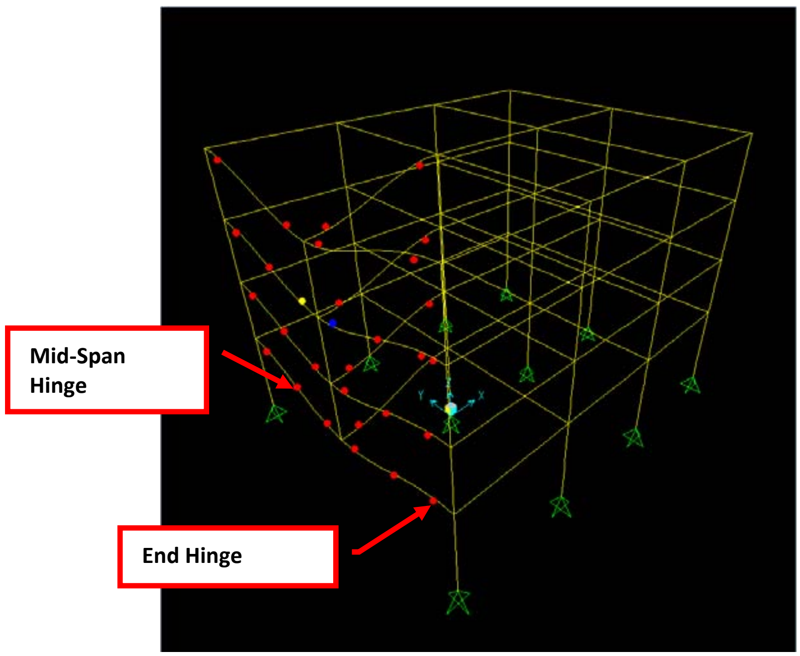

Figure 6. Hinge Locations

Each of the hinge locations shown in the figure above was defined using the modeling parameters provided in Table 4-1 in the UFC for RC beams. This table provides modeling parameters for non-linear hinges based on four different conditions as follows:

- Beams controlled by flexure
- Beams controlled by shear
- Beams controlled by inadequate development or splicing along the span
- Beams controlled by inadequate embedment into beam-column joint

Therefore, depending on the structural configuration, shear demand, and reinforcement ratios, each hinge in the model could have different modeling parameters. To illustrate the procedure for selecting the non-linear hinges, the calculations to determine the end and mid-span hinges for the perimeter beam in the N-S direction are presented next:

- End Hinges: The N-S spandrel is a beam controlled by flexure. Both end hinges in this case were similar because they have the same load, reinforcement, and geometry. First, it must be determined if the transverse reinforcement is conforming or nonconforming. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ *d/3*, and if, for components of moderate and high ductility demand, the strength provided by the hoops (*Vs*) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming (NC). The spandrel beam in this building has an effective depth (d) of approximately 28in. Since the stirrups are spaced at 9in (less than d/3), the end regions of the spandrel beam are considered as conforming. Next, the shear demand on the concrete portion of the beam at these end regions must be verified as follows:

Shear at beam ends	$V := 50.9\text{kip}$	*from comp. model with load case and column removed*
Concrete compressive strength	$f_c := 2.5\text{ksi} \cdot 1.5 = 3.75\text{ksi}$	
Beam depth	$d := 28\text{in}$	
Beam width	$b_w := 12\text{in}$	

$$\frac{V}{b_w \cdot d \cdot \text{psi}\sqrt{\dfrac{f_c}{\text{psi}}}} = 2.474 \qquad < 3.0$$

The last parameter needed to determine the hinge properties is the parameter (ρ- ρ'/ ρ_{bal}), where ρ is the reinforcement ratio of the bottom reinforcing steel and ρ' is the reinforcement ratio of top reinforcing steel. At the ends of the N-S spandrel beam, the area of top reinforcing steel is larger than the bottom reinforcing steel. Therefore, the parameter (ρ- ρ'/ ρ_{bal}) is less than 0.0, and the modeling parameters for the end hinges of the N-S spandrel beam are: a= 0.063, b=0.1 and c=0.2 per table 4-1 in the guidelines. The end hinges modeling force-deflection curve is illustrated in Figure 7.

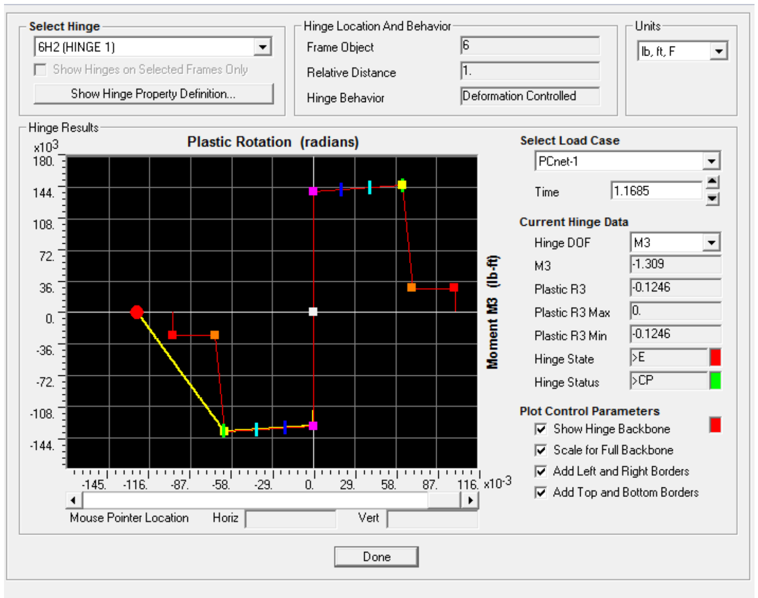

Figure 7. Spandrel Beam End Hinge Force-deformation Modeling Curve

- Mid-span Hinge: At mid-span there are no stirrups provided; therefore, the hinge region is determined to be NC. The shear demand is less than as calculated for the ends, which means that the shear parameter results in less than 3.0. The reinforcement ratio is as follows:

Bottom Steel

$$A_{b1} := 1.27\,in^2 + 1.56\,in^2 = 2.83\,in^2$$

$$\rho := \frac{A_{b1}}{12in \cdot 28in} = 0.00842 \qquad \beta_1 := 0.85$$

$$\rho_{bal} := 0.85 \left[\frac{87000psi \cdot \beta_1 \cdot 2500psi \cdot 1.5}{40000psi \cdot 1.25 \cdot (87000psi + 40000psi \cdot 1.25)} \right] = 0.034$$

$$\frac{\rho - 0}{\rho_{bal}} = 0.24$$

Therefore, based on the Table 4-1 in the UFC, the modeling hinge parameters for the mid-span hinge are: a=0.025, b= 0.03, c= 0.2 (Figure 8 below).

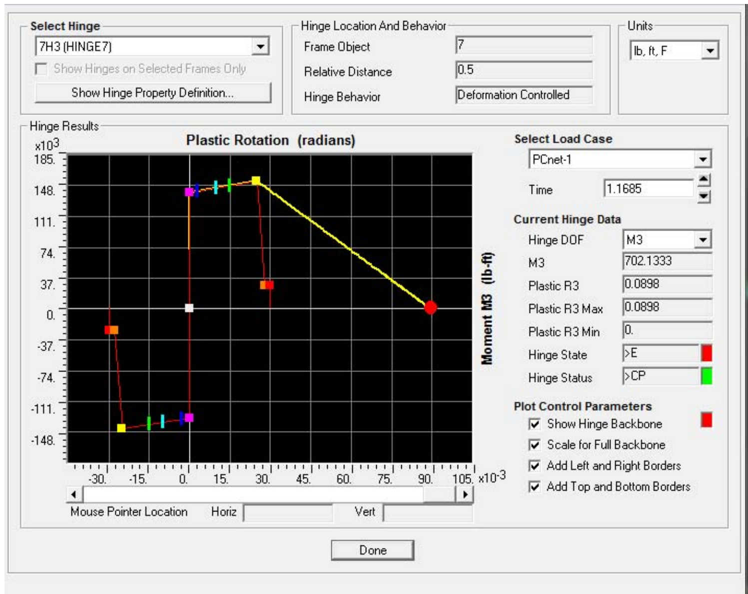

Figure 8. Spandrel Beam Mid-span Hinge Force-deformation Curve.

Using the approach explained above, modeling parameters for the non-linear hinges of every beam element were determined and assigned to their corresponding components in the computer model. It is important to point out that the hinge force-deformation curves shown below allow strain hardening of 5% at the point expected to be the maximum allowed rotation. This is less than the 10% maximum hardening allowed in ASCE 41. The reason for this difference is the larger allowable rotations used in progressive collapse analyses. The yield moment capacities in the curves shown below are calculated by the model based on the reinforcing steel and material properties assigned to each component.

Step 4 - Non-linear Dynamic Analysis Parameters

The dynamic analysis was performed using the “Nonlinear Direct Integration Time History” option in SAP 2000. The time integration method used was the Newmark method of integration and the Gamma and Beta parameters used corresponded to the default options in SAP2000 which generally provide good results and convergence times. Other analysis parameters included the damping ratio, time step, and column removal time. For this analysis, these parameters were taken as follows.

- Damping ratio = 1%
- Column removal and time step = 1/20 of the structure’s natural period
- Analysis Time Step = 1/200 of the structure’s natural period

The natural period of response was calculated to be 0.25 seconds. This was determined by performing a Modal Analysis, and selecting the Natural Period (T) of the dominating mode of vibration. The

dominating mode of vibration was selected visually based on the location of the column removal and the motion of the structure.

Step 5 – Instantaneous Column Removal

The final step to complete a NLD AP analysis is the instantaneous removal of the column or load bearing member. Depending on the software or analysis tool being used, the instantaneous removal of the column could be performed in several different ways. In SAP2000, the model does not allow the instantaneous removal of a structural component while performing a time history analysis. Therefore, in order to simulate the removal of the column, a series of steps are required in which the column to be removed is replaced by equivalent superimposed forces that are then removed over time using a linear function. A more detailed explanation of each of these steps follows.

First, a linear static analysis was performed using the un-factored extreme load case to calculate the reactions at the top joint of the column to be removed. Then, the column was removed from the model and the calculated reactions are applied at the column joint. After the columns has be substituted with equivalent reaction forces, a new linear static analysis was performed, and the resulting flexural moments diagrams and deflections were compared to the results obtained from the initial linear static analysis that included the column. Both linear-static analyses (with the column and with substitute reactions) resulted in identical moment diagrams and deflections. Therefore, the column was successfully replaced by equivalent superimposed reaction forces. This is illustrated in Figure 9 below.

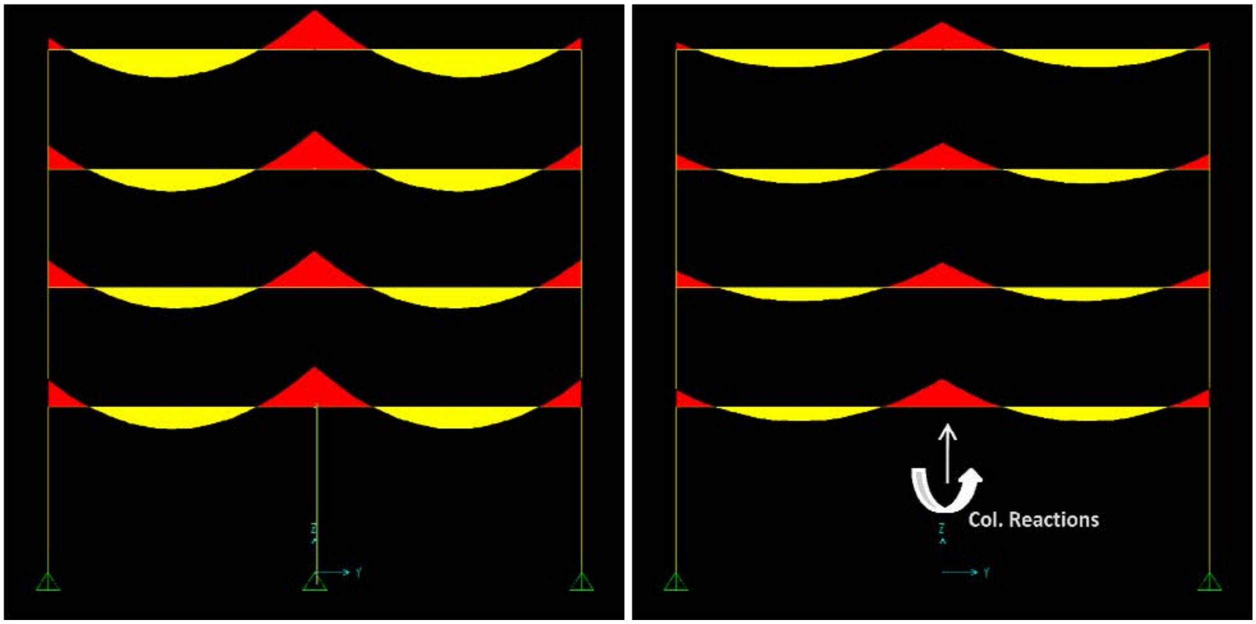

Figure 9. Moment Diagrams, (w/column left, w/sub. reactions right)

Next, using the model with the removed column and the equivalent reactions forces applied at the column joint, a dynamic non-linear analysis case was set up using the time history option in SAP2000. To simulate the instantaneous removal of the column, the equivalent reaction loads were removed over time using a ramping function as shown in the figure below. For this analysis, the removal of the load was performed over a small period of time equal to 1/20 of the natural period of the structure. This ensures that enough sample data points are obtained during the removal of the column.

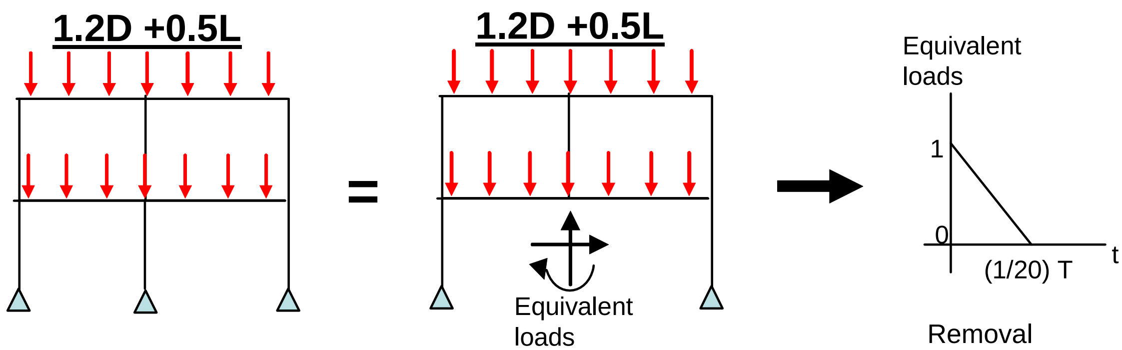

Figure 10. Instantaneous Column Removal

After the equivalent column loads were removed, the building was allowed to deform until it settled or a number of hinges failed, and the maximum plastic rotation was recorded for all hinges formed during the analyses.

Summary of Results and Comparison to LS approach

The results of the analysis for this column removal showed that the existing structural components are able to resist progressive collapse. As seen in Figure 11 the structure deflected approximately 4 inches before settling. Several hinge locations reached the plastic deformation zone, but all remained within

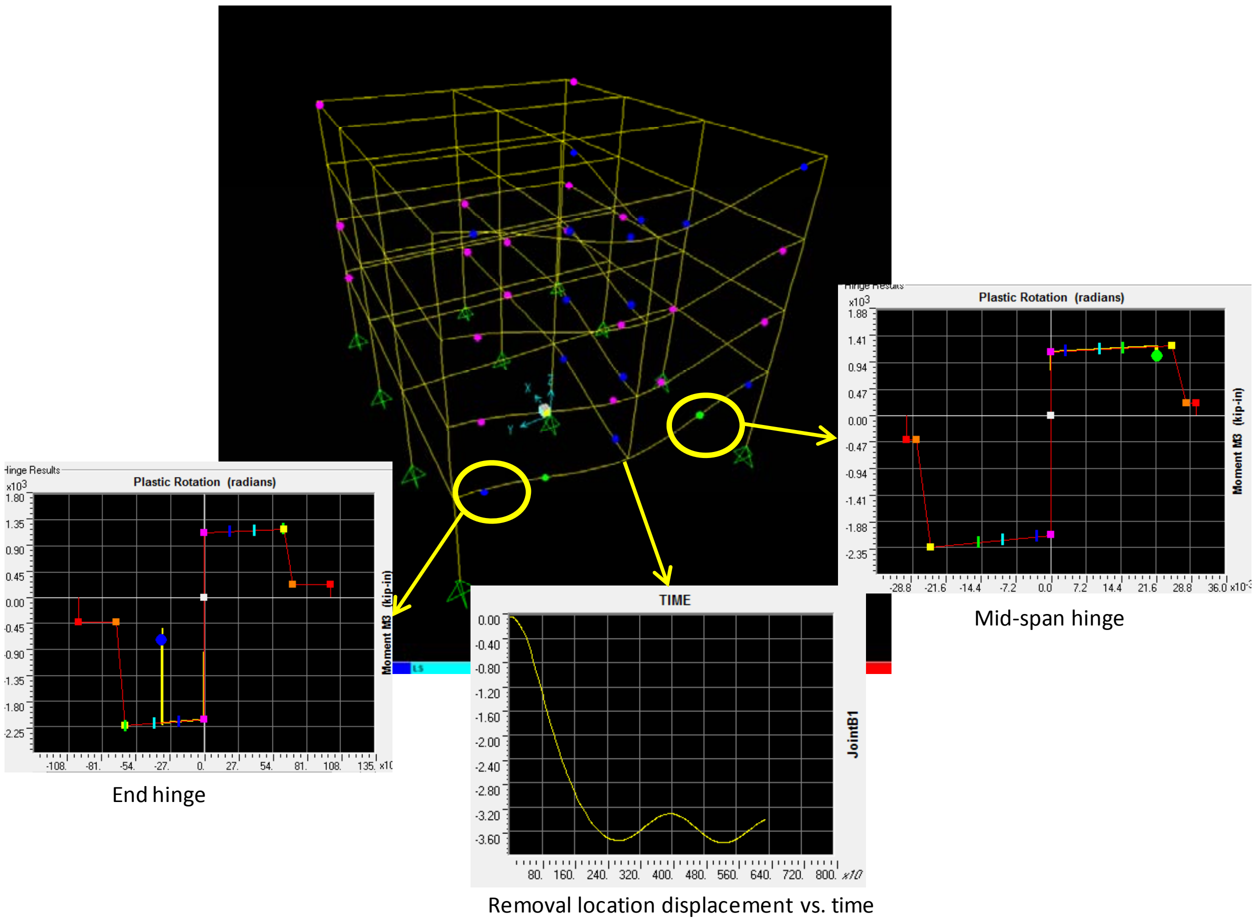

Figure 11 NLD Analysis Completed

In order to demonstrate the efficiency of the NLD AP approach, an LS AP analysis was performed using UFC 4-023-03 prescribed procedures for the same column removal. In the LS AP approach, the applied load is enhanced by a “Load Increase Factor” that approximately accounts for both inertial and nonlinear effects. The enhanced load is applied to the linear static model that has been modified by removal of a column and the calculated internal member forces (actions) due to the enhanced loads are compared to the expected member capacities. The LS analysis was performed only for deformation-controlled actions (UFC also requires to check force-controlled actions) using a calculated load increase factor (LIF/ *m*) of 1.33. The *m* factor used to determine the effective multiplier was determined from Table 4-2 in the UFC.

The results of the LS AP approach showed that the building is not adequate to resist progressive collapse. In addition, the LS model showed that increases of reinforcing steel area ranging from 2 to 3 were required for the spandrel beams and portions of the column strip. Figure 12 below shows the reinforcing steel requirements calculated with LS approach at the second floor. More discussion regarding the difference between the results of obatined with the LS and the NLD AP analyses and possible reasons for this discrepancy is provided below.

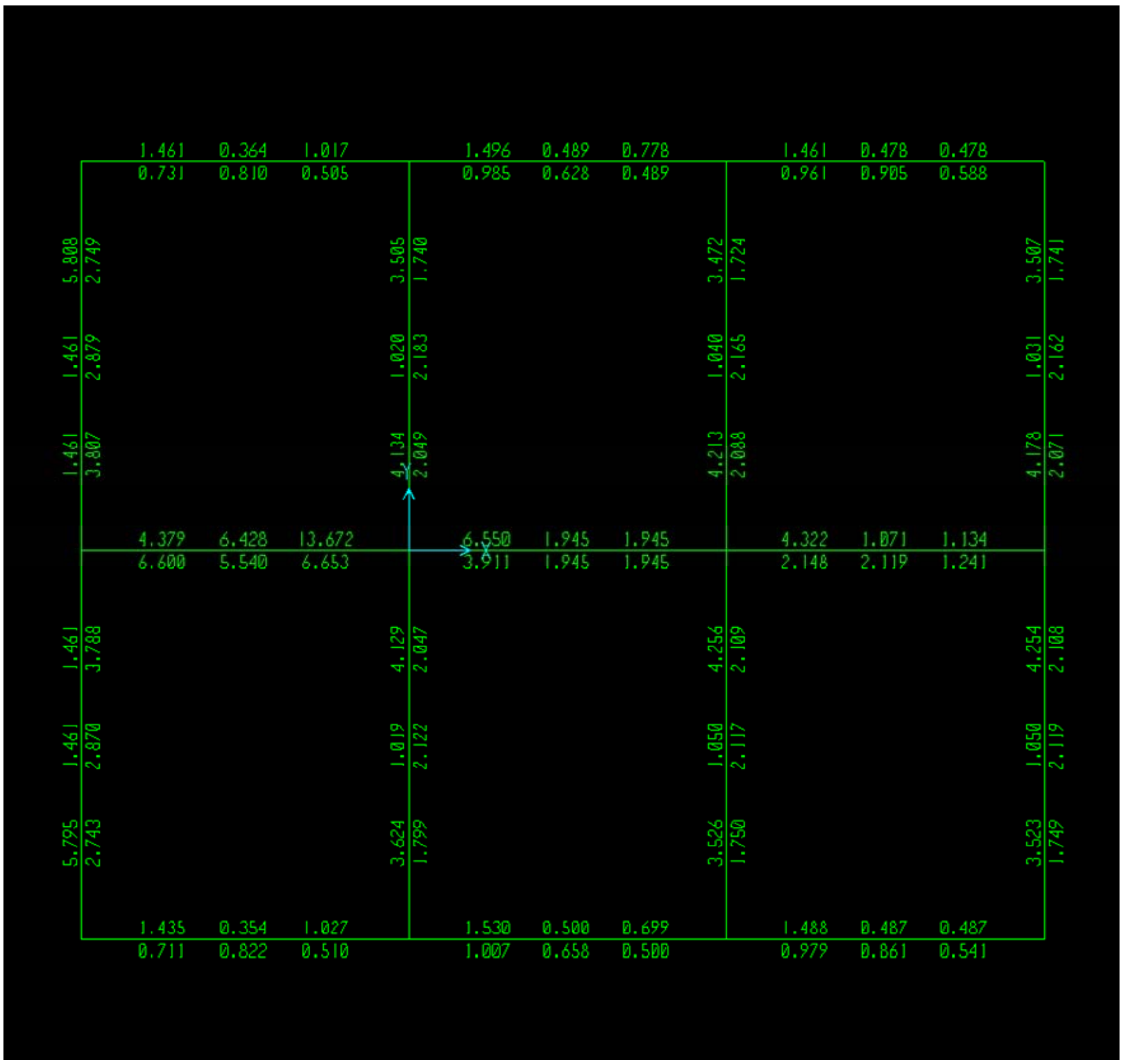

Figure 12. LS AP Analysis Results

Conclusions

Significantly different results were obtained using the NLD and LS analysis approaches. The NLD analysis resulted in deformations within the limits prescribed by the UFC, thus confirming that the building is adequate to resist progressive collapse. Conversely, the LS analysis approach showed that an upgrade would be required to bring the building into compliance for progressive collapse.

The principal reason for the acceptable performance of the structure when using the NLD approach was the capacity provided by the column strip portion of the slab system. There is a considerable amount of steel reinforcement properly detailed in these areas of the slab, which helps to reduce the overall deflection of the structure. This is important because the contribution of the floor slabs is neglected (as required by the UFC) in the lower fidelity LS analyses. This was verified in a separate analysis (not cover here) in which the ends of the column strips elements were “released” to not transfer moment forces and the structure underwent significant deformation and a number of hinges exceeded the rotation limits.

The comparison presented in this paper showed that NLD can be very effective for AP analysis of existing buildings were implementation of retrofits and upgrades are expensive. One of the reasons why the NLD is more effective than the LS AP procedure is that it takes advantage of all components of the structure that are capable of deforming plastically and absorbing energy. The LS approach is limited by allowable detail (secondary elements such as slabs are not permitted in those analyses) and by more restrictive stress demands which result in stiffer and heavier structural requirements. Another reason why the LS approach produces more conservative designs is the use of load increase factors to account for inertial and non-linear effects. These factors are applied to all components around the loss locations, but are based on the most critical (“weakest”) primary component around the loss location. Therefore, the most restrictive component controls to overall analysis.

Finally, is important to point out that the example presented in this paper focused only on one column removal and the related deformation controlled actions. A complete progressive collapse analysis per UFC 4-023-03 requires several other types of analyses and checks such as force-controlled actions and adequacy of secondary components, which were not included in this example.

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Non-Linear Dynamic Alternate Path Analysis for Progressive Collapse: Detailed Procedures Using UFC 4-023-03 (revised July 2009)

Aldo E. McKay, P.E

34th DDESB Seminar – Portland, Oregon – July 13 -15, 2010

Outline

- Progressive Collapse/Alternate Path (AP) Analysis
- Background of UFC 4-023-03
- Present detailed example of application of the Non-Linear Dynamic (NLD) Alternate Path (AP) approach per the July 2009 UFC 4-023-03
- Comparisons between Linear Static (LS) and Non-Linear Dynamic Alternate Path approaches
    - Advantages of Non-Linear Dynamic approach

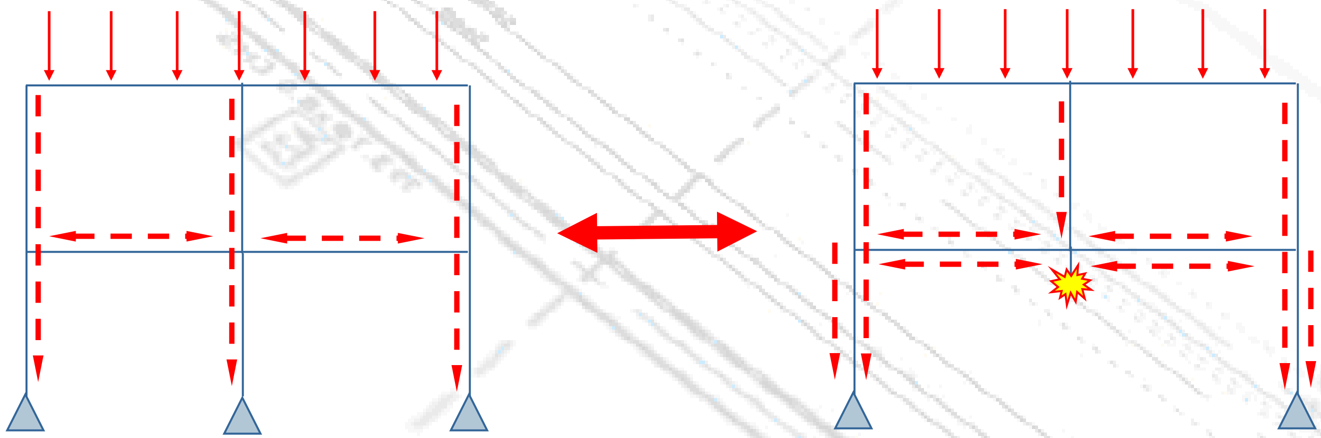
Progressive Collapse

Murrah Federal Building

World Trade Center

Alternate Path Analysis

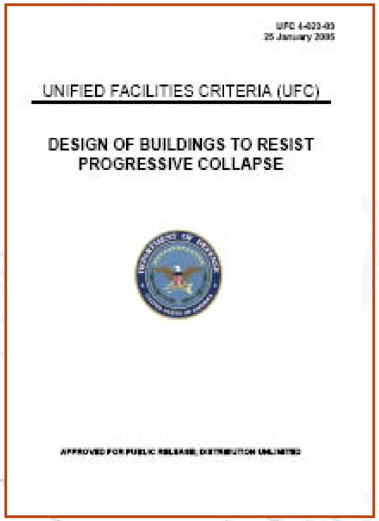
- Structure must bridge over missing element
- Resulting damage less than acceptance criteria
- Three analytical procedures may be used
  - Nonlinear Dynamic (NLD) – (more time consuming and complex)
  - Nonlinear Static (NLS)
  - Linear Static (LS) – (simpler and faster to perform)



UFC 4-023-03 Differences between 2005 and 2009 Versions

- 4 years since UFC 4-023-03 was first published: various omissions, ambiguities and opportunities for improvement have been identified by civilian and government users

- New version initiated in 2006 and published in 2009 includes several improvements:
  - Use of Occupancy Categories similar to ASCE 7 (Previously used Military Definitions of LOP)
  - Indirect method (Tie forces) modified to enhanced load redistribution capacity
  - Load factors for static approached in AP derived from research studies
  - Structural response criteria specified in terms of deformation/Force controlled actions derived from ASCE 41-06
  - Added a non-threat specific, local hardening procedure (ELR)


UFC 4-023-03
25 January 2005

UNIFIED FACILITIES CRITERIA (UFC)

DESIGN OF BUILDINGS TO RESIST PROGRESSIVE COLLAPSE

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Why ASCE 41 approach was adopted?

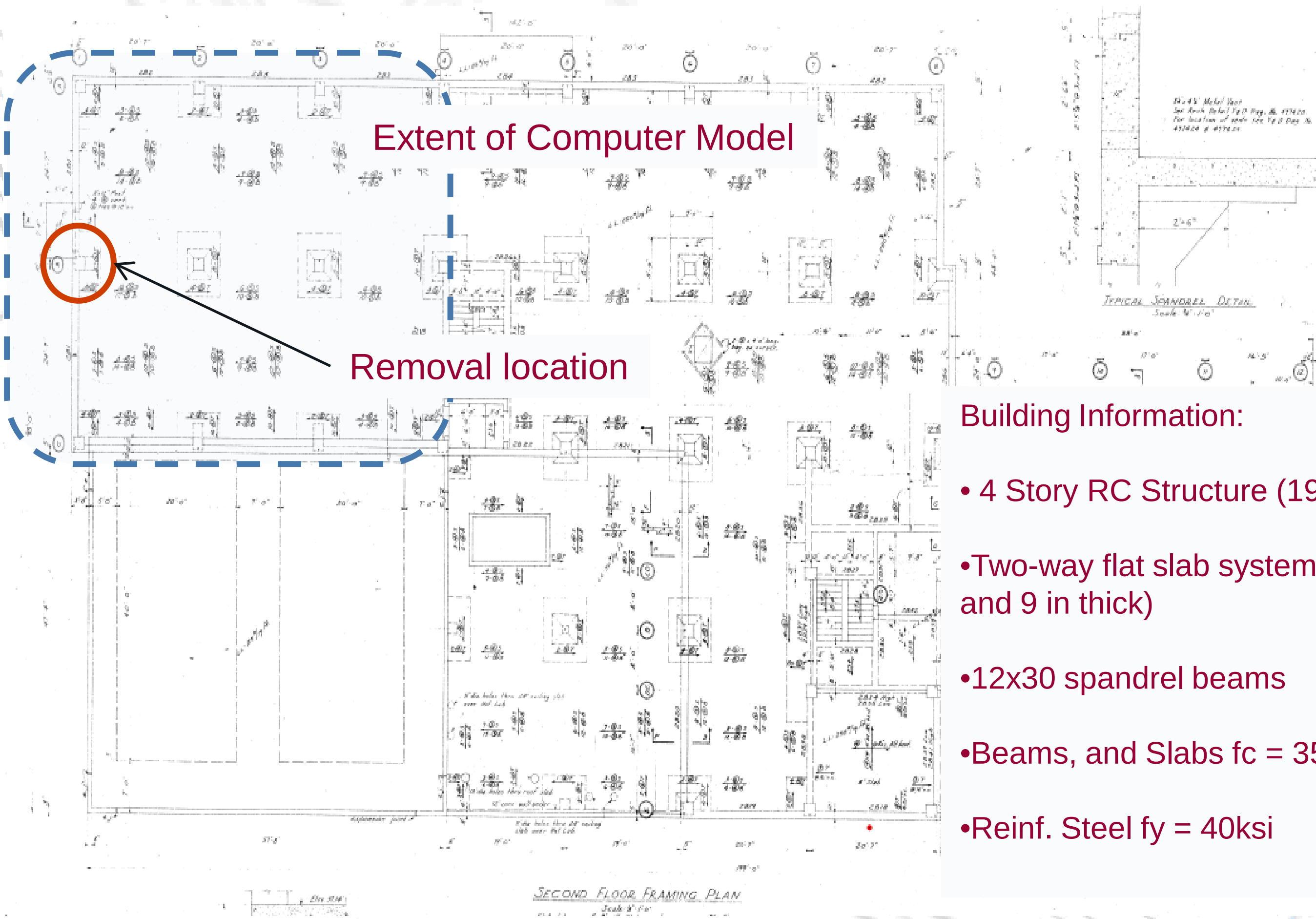
- ASCE 41 and UFC 4-023-03 deal with extreme events that severely damage structures which must not collapse or otherwise imperil the occupants.
- The ASCE 41 methodology was developed and vetted by a panel of structural engineering experts over many years of effort and could be modified in a straightforward manner for progressive collapse design.
- Five materials are considered: steel, RC, masonry, wood, and cold formed steel, in ASCE 41 and UFC 4-023-03.
- Explicit requirements and guidance for analyzing and designing multiple building types for each material are provided in ASCE 41.
- Careful attention is given in ASCE 41 to deformation- and force-controlled actions, as well as primary and secondary components.
- The acceptance criteria and modeling parameters in ASCE 41 can be scaled for different structural performance levels.

Analysis Procedure per UFC 4-023-03

- Un-factored Extreme Load Case (1.2D+0.5L)
- Computer Model must allow modeling of dynamic and non-linear behavior
- Structure reaches equilibrium, then column is instantaneously removed
- Resulting maximum deformations are compared to acceptance criteria

NLD AP Analysis Example

- Building Description – DoD OC II Building Renovation

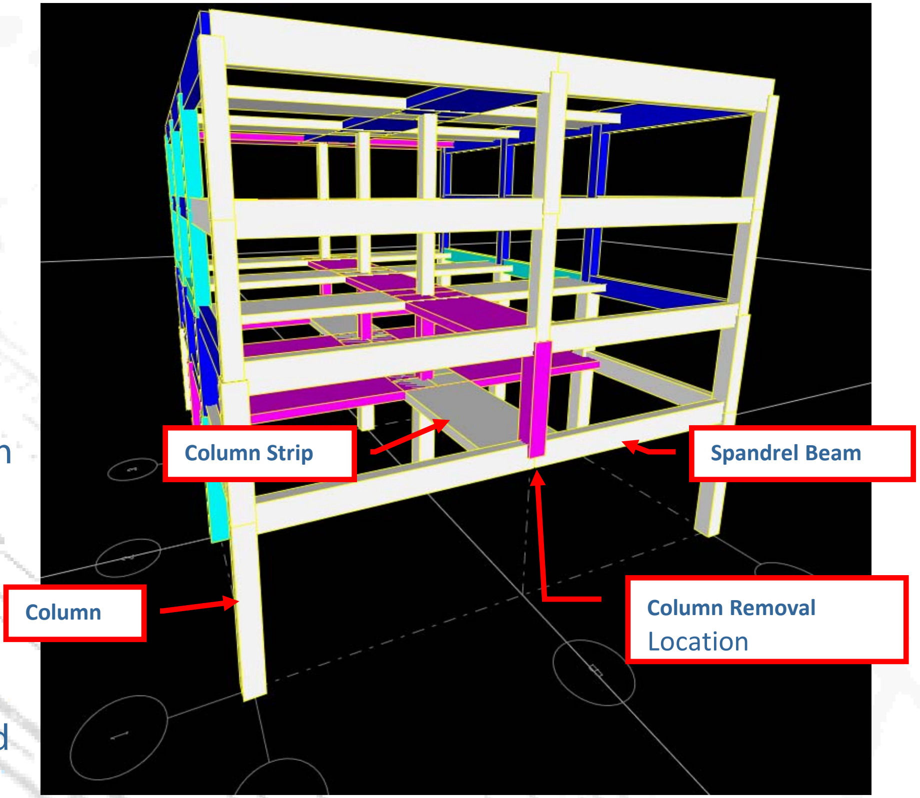


Building Information:

- 4 Story RC Structure (1940's)
- Two-way flat slab system (11in and 9 in thick)
- 12x30 spandrel beams
- Beams, and Slabs fc = 3500psi
- Reinf. Steel fy = 40ksi

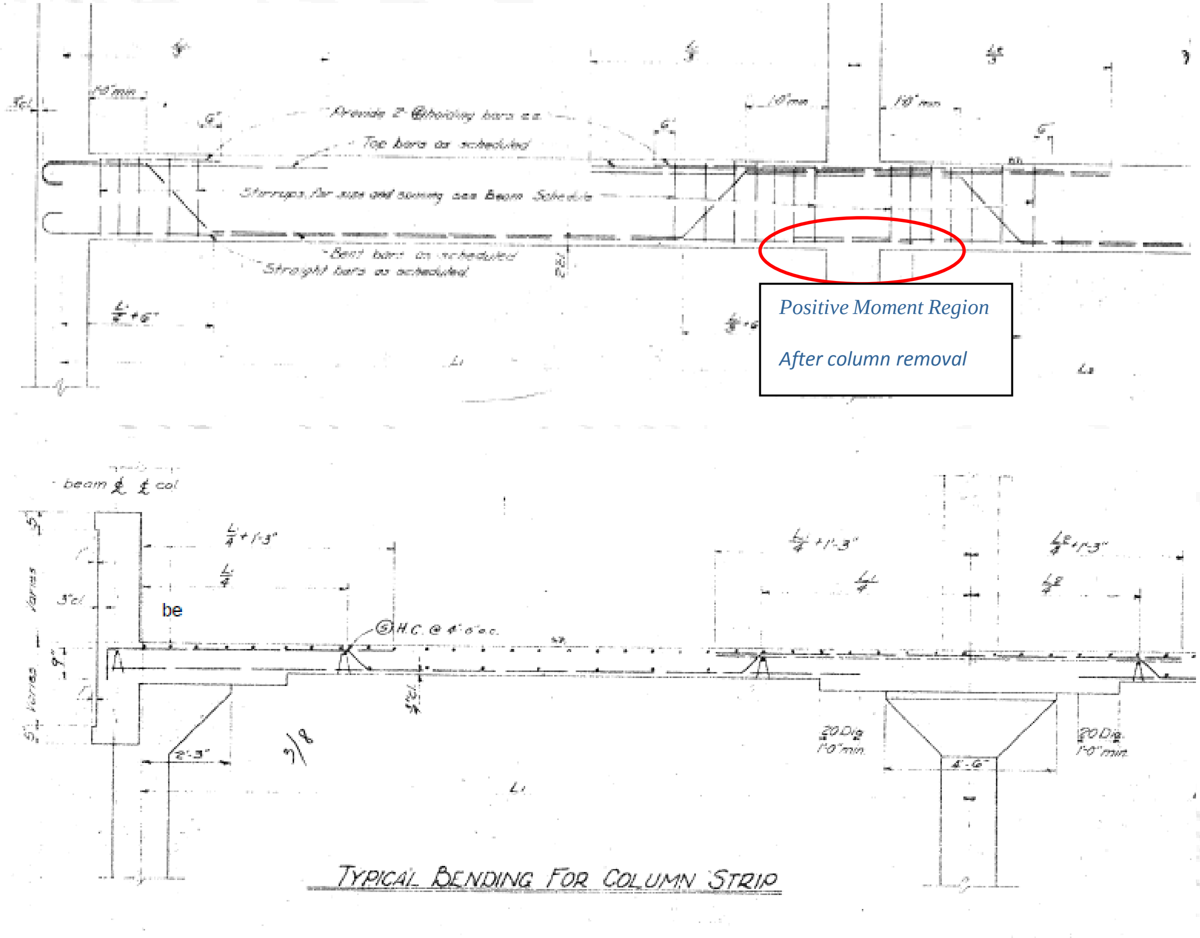
NLD AP Analysis Example

- Computer Model
  - SAP2000 – dynamic loads and nonlinear behavior capable
  - Only North-East portion of building modeled
  - Primary members:
    - Spandrel Beams
    - Slab Column Strip (As beam element)
    - Columns (pinned at the bottom)
  - Secondary members not included per UFC



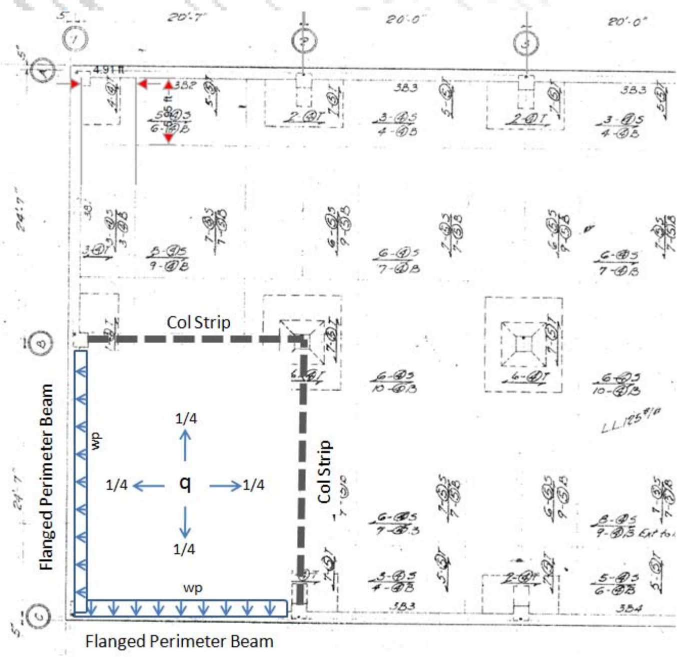
NLD AP Analysis Example

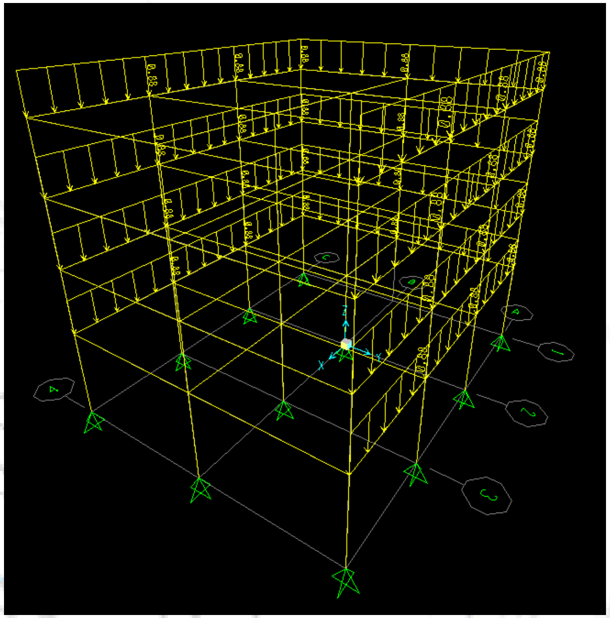
- Computer Model, cont'd – Interpretation of Structural Drawings



NLD AP Analysis Example

- Loads and Mass
  - Loads applied as linear loads
  - Separate cases for each type: LL, DL, etc.
  - Dead loads include self-weight
  - Mass source for dynamic system from added loads not from Elements





NLD AP Analysis Example

- Non-Linear Hinges

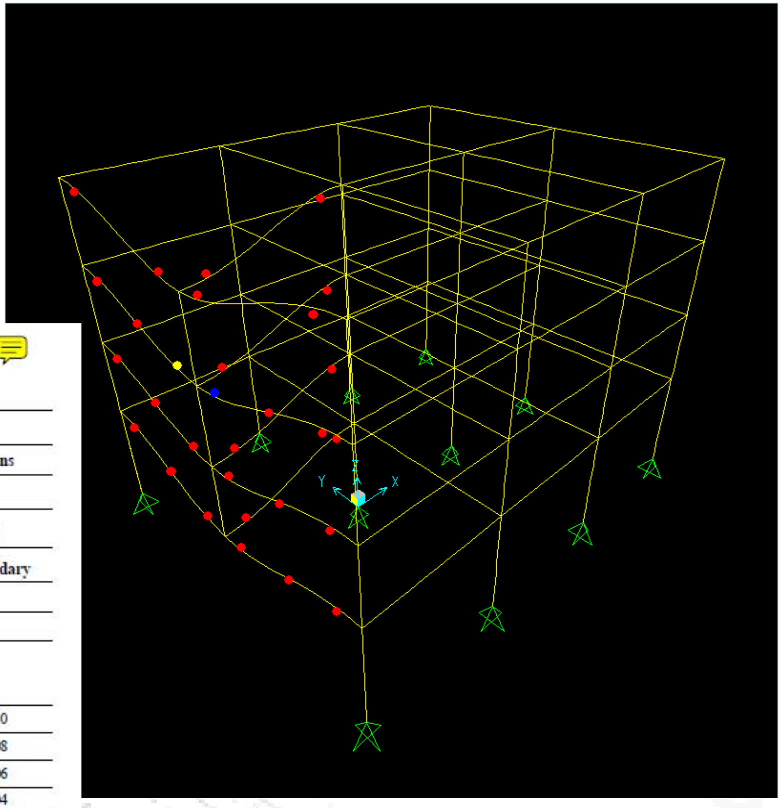


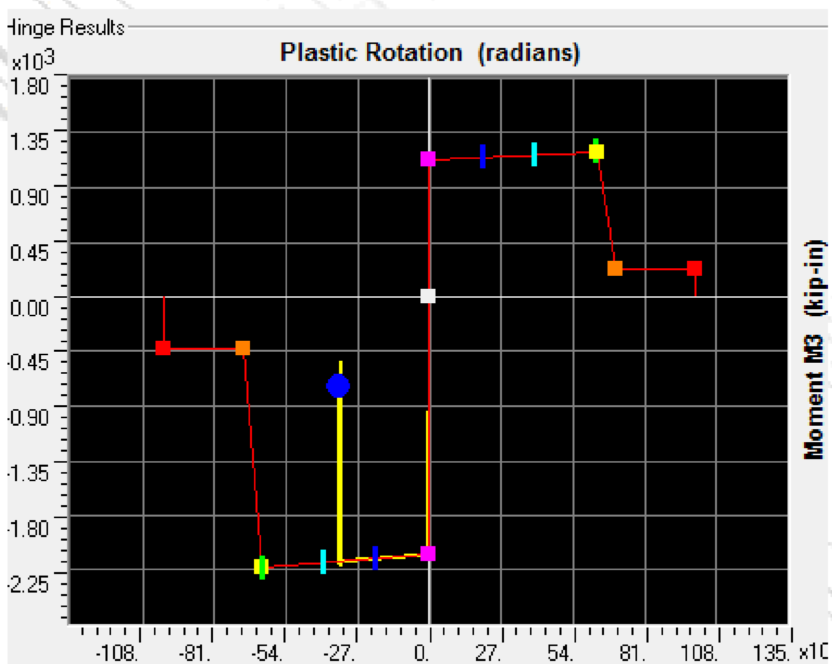
Table 4-1. Nonlinear Modeling Parameters and Acceptance Criteria for Reinforced Concrete Beams (Replacement for Table 6-7 in ASCE 41)

Conditions			Modeling Parameters[1]			Acceptance Criteria[1,2]		
			Plastic Rotations Angle, radians		Residual Strength Ratio	Plastic Rotations Angle, radians		
							Component Type	
							Primary	Secondary
			a	b	c			
i. Beams controlled by flexure[3]								
$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinf.[4]	$\frac{V}{b_w d\sqrt{f'_c}}$						
≤ 0.0	C	≤ 3	0.063	0.10	0.2		0.063	0.10
≤ 0.0	C	≥ 6	0.05	0.08	0.2		0.05	0.08
≥ 0.5	C	≤ 3	0.05	0.06	0.2		0.05	0.06
≥ 0.5	C	≥ 6	0.038	0.04	0.2		0.038	0.04
≤ 0.0	NC	≤ 3	0.05	0.06	0.2		0.05	0.06
≤ 0.0	NC	≥ 6	0.025	0.03	0.2		0.025	0.03
≥ 0.5	NC	≤ 3	0.025	0.03	0.2		0.025	0.03
≥ 0.5	NC	≥ 6	0.013	0.02	0.2		0.013	0.02
ii. Beams controlled by shear[3]								
Stirrup spacing ≤ d/2			0.0030	0.02	0.2		0.002	0.01
Stirrup spacing > d/2			0.0030	0.01	0.2		0.002	0.005
iii. Beams controlled by inadequate development or splicing along the span[3]								
Stirrup spacing ≤ d/2			0.0030	0.02	0.0		0.002	0.01
Stirrup spacing > d/2			0.0030	0.01	0.0		0.002	0.005
iv. Beams controlled by inadequate embedment into beam-column joint[3]								
			0.015	0.03	0.2		0.01	0.02

NLD AP Analysis Example

- Non-Linear Hinges, cont'd – example for N-S spandrel Beam
  - End Hinge:
    - Depth (d) approx. 28in; stirrups at 9 in. O.C < d/3 – End region is Conforming (C)
    - At the ends, negative steel is more than positive steel. Therefore, the parameter ρ – ρ' / ρbal < 0.0
  - Finally the shear ratio parameter:

$$\frac{V}{b_w \cdot d \cdot psi \sqrt{\frac{f_c}{psi}}} = 2.474 \quad < 3.0$$

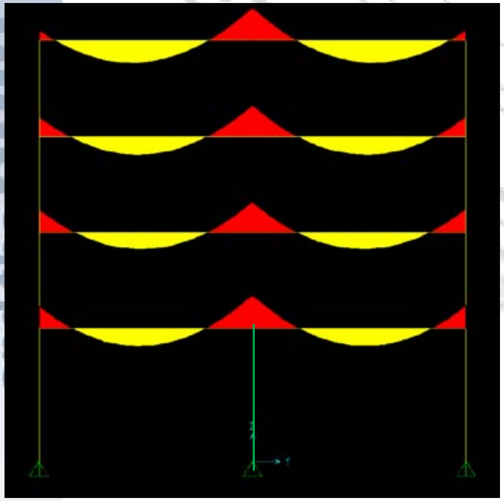


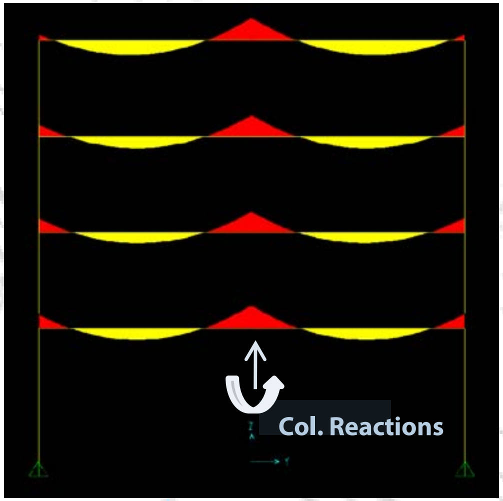
NLD AP Analysis Example

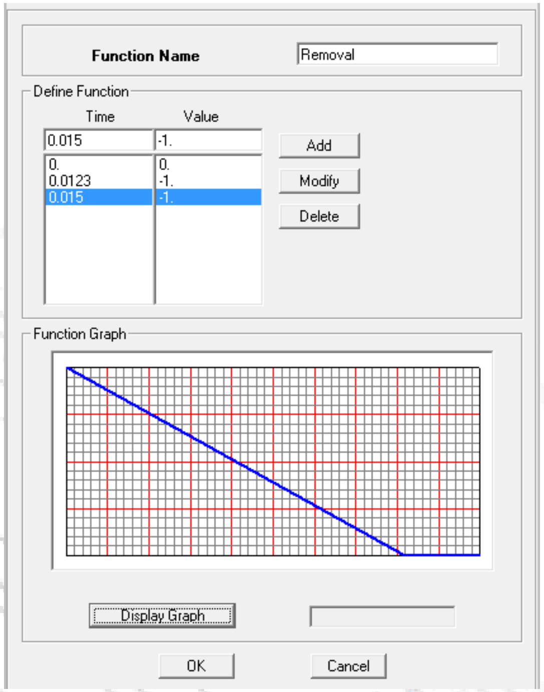
- Non-linear Dynamic Parameters for analysis
  - Natural Period (T) = 0.25 seconds, calculated from Modal Analysis
  - Damping ratio = 1%
  - Analysis Step = 1/200 of T
  - Use P-delta effects and time history analysis in SAP2000

NLD AP Analysis Example

- Instantaneous Removal
  - Different procedure depending on structural software used
  - SAP2000 doesn’t allowed elements deletion while performing a time history analysis.
  - Therefore, a series of steps are required to simulate the removal of the column

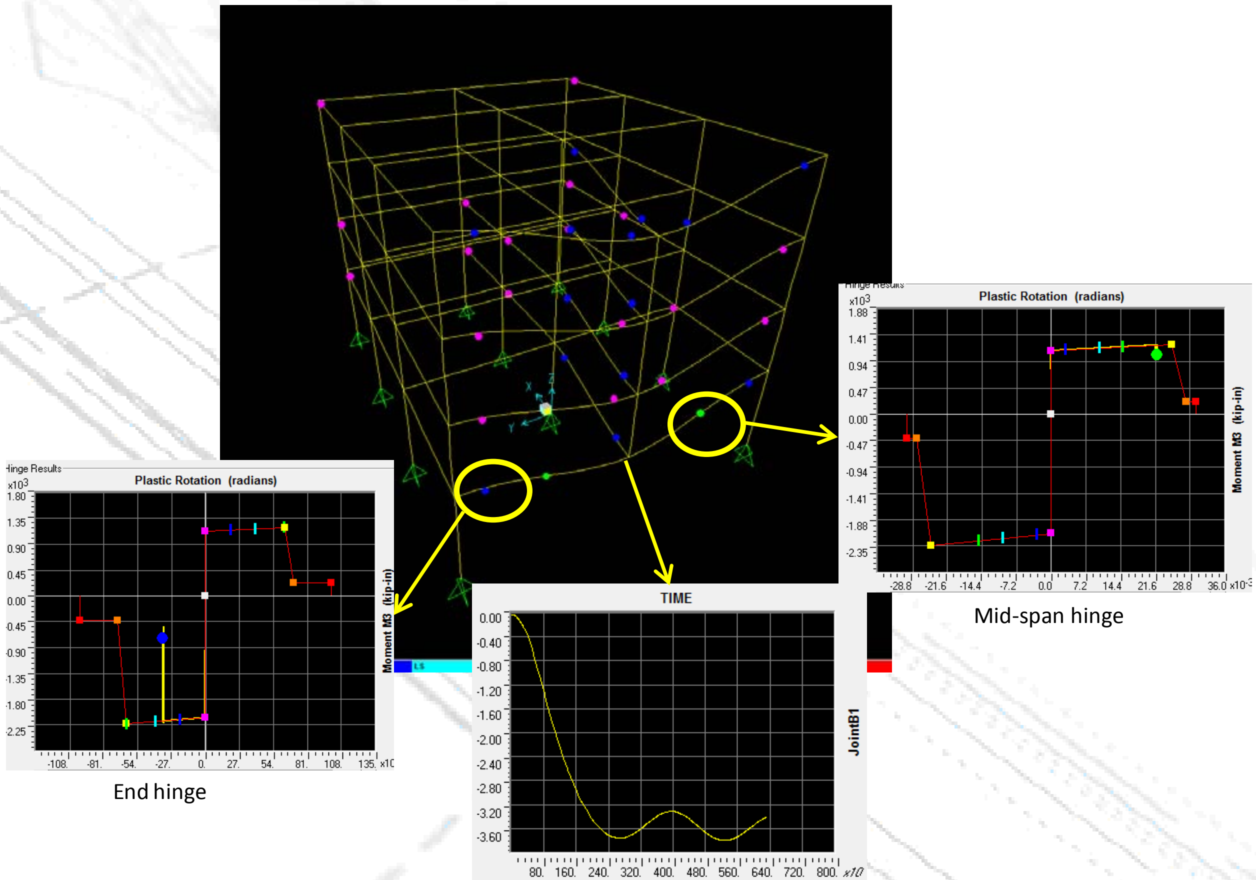






NLD AP Analysis Example

- Results
  - No Hinges failed, Max deflection approximately 4 inches



Mid-span hinge

End hinge

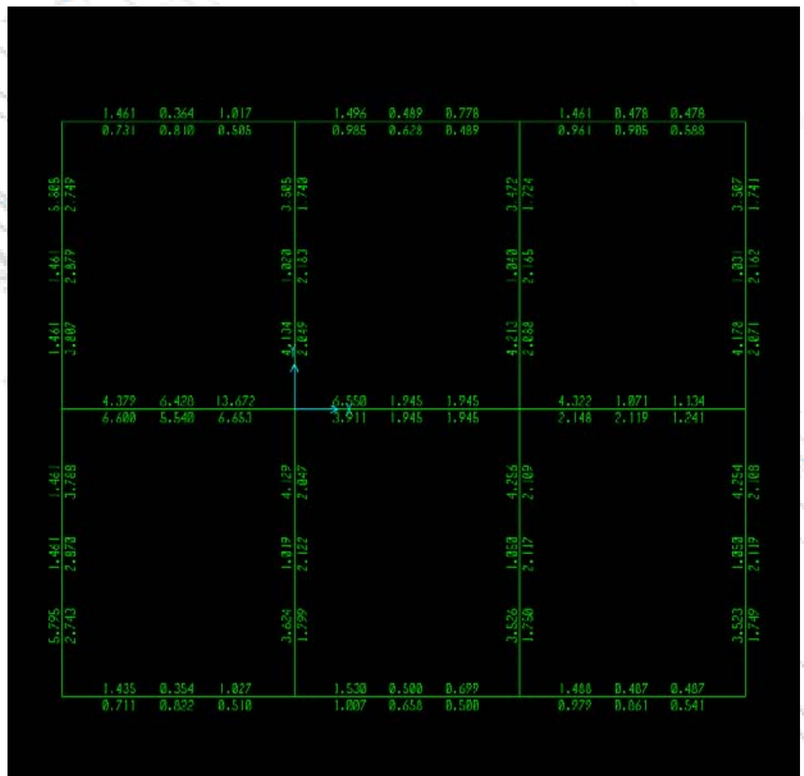
Removal location displacement vs. time

Linear Static AP Analysis

- Performed per requirements in July 2009, UFC 4-023-03
  - Applied Load is enhanced to account for Non-linear and inertial effects
  - Enhanced load applied to building model with column removed
  - Calculated member forces are compared to expected capacities
  - Only Force-controlled actions compared to NLD results
  - Effective load multiplier used for analysis = 1.33

Linear Static AP Analysis

- Results
  - Member capacities exceeded in several locations of spandrel and column strip
  - Increase factors of reinforcing steel required for building to work ranging from 2 to 3 generally



Conclusions

- Significantly different results were obtained using the NLD and LS analysis approaches.
  - The NLD analysis resulted in deformations within the limits prescribed by the UFC, thus confirming that the building is adequate to resist progressive collapse.
  - Conversely, the LS analysis approach showed that an upgrade would be required to bring the building into compliance for progressive collapse.
- Colum strip of slabs added significant capacity to the system
  - It was properly detailed
  - This contribution if often neglected in lower fidelity LS analyses
- NLD AP approach can be very effective specially for retrofit of existing buildings where upgrades can be expensive
  - NLD takes advantage of the structure's capacity to deform plastically and absorbed energy
- LS AP approach is limited by:
  - restrictive stress demand which result in heavier structural requirements
  - Load factors are applied to all components around the loss locations, but are based on the most critical ("weakest") primary component around the loss location. Therefore, the most restrictive component controls the overall analysis.

Thank you, Questions?

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